The background of the cover is a photograph of a construction site at sunset. The sky is a vibrant orange and yellow, with silhouettes of construction cranes and the skeletal frame of a building under construction. The sun is low on the horizon, creating a bright lens flare effect. The overall mood is industrial and dramatic.

HANDBOOK OF **STRUCTURAL ENGINEERING** SECOND EDITION

Edited by
W.F. CHEN
E.M. LUI

Handbook of

**STRUCTURAL
ENGINEERING**

Handbook of

STRUCTURAL ENGINEERING

Edited by

**WAI-FAH CHEN
ERIC M. LUI**



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Abstract

This book is an encapsulation of a myriad of topics of interest to engineers working in the structural analysis, design, and rehabilitation fields. It is a comprehensive reference work and resource book written for advanced students and practicing engineers who wish to review standard practices as well as to keep abreast of new techniques and practices in the field of structural engineering. The *Handbook* stresses professional applications and includes materials that are presented in an easy-to-read and ready-to-use form. It contains many formulas, tables, and charts that give immediate answers to questions arising from practical work. The book covers not only traditional but also novel and innovative approaches to analysis, design, and rehabilitation problems.

Preface

The primary objective of this new edition of the *CRC Handbook of Structural Engineering* is to provide advanced students and practicing engineers with a useful reference to gain knowledge from and seek solutions to a broad spectrum of structural engineering problems. The myriad of topics covered in this handbook will serve as a good resource for readers to review standard practice and to keep abreast of new developments in the field.

Since the publication of the first edition, a number of new and exciting developments have emerged in the field of structural engineering. Advanced analysis for structural design, performance-based design of earthquake resistant structures, life cycle evaluation, and condition assessment of existing structures, the use of high-performance materials for construction, and design for fire safety are some examples. Likewise, a number of design specifications and codes have been revised by the respective codification committees to reflect our increased understanding of structural behavior. All these developments and changes have been implemented in this new edition. In addition to updating, expanding, and rearranging some of the existing chapters to make the book more informative and cohesive, the following topics have been added to the new edition: fundamental theories of structural dynamics; advanced analysis; wind and earthquake resistant design; design of prestressed concrete, masonry, timber, and glass structures; properties, behavior, and use of high-performance steel, concrete, and fiber-reinforced polymers; semirigid frame structures; life cycle evaluation and condition assessment of existing structures; structural bracing; and structural design for fire safety. The inclusion of these new chapters should enhance the comprehensiveness of the handbook.

For ease of reading, the chapters are divided into six sections. Section I presents fundamental principles of structural analysis for static and dynamic loads. Section II addresses deterministic and probabilistic design theories and describes their applications for the design of structures using different construction materials. Section III discusses high-performance materials and their applications for structural design and rehabilitation. Section IV introduces the principles and practice of seismic and performance-based design of buildings and bridges. Section V is a collection of chapters that address the behavior, analysis, and design of various special structures such as multistory rigid and semirigid frames, short- and long-span bridges, cooling towers, as well as tunnel and glass structures. Section VI is a miscellany of topics of interest to structural engineers. In this section are included materials related to connections, effective length factors, bracing, floor system, fatigue, fracture, passive and active control, life cycle evaluation, condition assessment, and fire safety.

Like its previous edition, this handbook stresses practical applications and emphasizes easy implementations of the materials presented. To avoid lengthy and tedious derivations, many equations, tables, and charts are given in passing without much substantiation. Nevertheless, a succinct discussion of the essential elements is often given to allow readers to gain a better understanding of the underlying theory, and many chapters have extensive reference and reading lists and websites appended at the end for engineers and designers who seek additional or more in-depth information. While all chapters in this handbook are meant to be sufficiently independent of one another, and can be perused without first having proficiency in the materials presented in other chapters, some prerequisite knowledge of the fundamentals of structures is presupposed.

This handbook is the product of a cumulative effort from an international group of academicians and practitioners, who are authorities in their fields, graciously sharing their extensive knowledge and invaluable expertise with the structural engineering profession. The authors of the various chapters in

this handbook hail from North America, Europe, and Asia. Their scientific thinking and engineering practice are reflective of the global nature of engineering in general, and structural engineering in particular. Their participation in this project is greatly appreciated. Thanks are also due to Cindy Carelli (acquisitions editor), Jessica Vakili (project coordinator), and the entire production staff of CRC Press for making the process of producing this handbook more enjoyable.

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List of Abbreviations

2D	two-dimensional	CFM	Continuous filament materials
AASHTO	American Association of State Highway and Transportation Officials	CFRP	Carbon fiber-reinforced plastic
ACI	American Concrete Institute	CGSB	Canadian General Standards Board
ACMA	American Composites Manufacturers Association	CHS	Circular hollow section
ADAS	Added damping and stiffness	CIB	Conseil International du Batiment
ADRS	Acceleration-displacement response spectrum	CIDECT	Comité International pour le Developement et l'Etude de la Construction Tubulaire
AISC	American Institute of Steel Construction	CIDH	Cast-in-drilled-hole
AISI	American Iron and Steel Institute	CLT	Classical lamination theory
ANSI	American National Standards Institute	COV	Coefficient of variation
APA	American Plywood Association	CQC	Complete-quadratic-combination
AREMA	American Railway Engineering and Maintenance-of-way Association	CRC	Column Research Council
ARS	Acceleration response spectra	CS	Condition state
AS	Aerial spinning	CSA	Canadian Standards Association
ASCE	American Society of Civil Engineers	CSM	Capacity spectrum method
ASD	Allowable stress design	CTOD	The crack tip opening displacement test
ASME	American Society of Mechanical Engineers	CUREE	Consortium of Universities for Research in Earthquake Engineering
ASTM	American Society of Testing and Materials	CVN	Charpy V-Notch
ATC	Applied Technology Council	DBE	Design basis earthquake
AWS	American Welding Society	DE	Design earthquake
BBC	Basic Building Code	DEn	Department of Energy
BIA	Brick Industry Association	DMM	Deep Mixing Method
BOCA	Building Officials and Code Administrators	DOF	Degree-of-freedom
BOEF	Beam on elastic foundation approach	DOT	Department of Transportation
BSI	British Standards Institution	DSP	Densified small particle
BSO	Basic safety objective	EBF	Eccentrically braced frame
BSSC	Building Seismic Safety Council	EC3	Eurocode 3
CABO	Council of American Building	ECCS	European Coal and Steel Community
CAFL	Constant-amplitude fatigue limit	ECS	European Committee for Standardization
CALREL	CAL-RELIability	ECSSI	Expanded Clay, Shale and Slate Institute
CBF	Concentrically braced frames	EDA	Elastic dynamic analysis
CDF	Cumulative distribution function	EDCH	Eurocomp Design Code and Handbook
CEB	Comité Eurointernationale du Béton	EDP	Engineering demand parameter
CFA	Composite Fabricators Association	EDR	Energy dissipating restraint
		EDWG	Energy Dissipation Working Group
		EERI	Earthquake Engineering Research Institute

ELF	Equivalent lateral force	IMF	Intermediate moment frame
EMC	Equilibrium moisture content	IMI	International Masonry Institute
EMS	European Macroseismic Scale	IO	Immediate occupancy
EOF	End one-flange	IOF	Interior one-flange
EPA	Effective peak acceleration	IRC	Institute for Research in Construction
EPB	Earth pressure balance	ISA	Inelastic static analysis
EPTA	European Pultrusion Technology Association	ISO	International Standard Organization
EPV	Effective peak velocity	ITF	Interior two-flange
ERS	Earthquake resisting system	JMA	Japan Meteorological Agency
ERSA	Elastic response spectrum analysis	JRA	Japan Road Association
ESA	Equivalent static analysis	JSME	Japan Society of Mechanical Engineers
ESDU	Engineering Sciences Data Unit	LA	Linear analysis
ETF	End two-flange	LAST	Lowest anticipated service temperature
FCAW	Flux-cored arc welding	LCADS	Life-Cycle Analysis of Deteriorating Structures
FCAW-S	Self-shielded flux-cored arc welding	LCR	Locked-coil rope
FEE	Functional evaluation earthquake	LDP	Linear dynamic procedure
FEM	Finite element model	LFRS	Lateral force resisting system
FEMA	Federal Emergency Management Agency	LRFD	Load and resistance factor design
FHWA	Federal Highway Administration	LSD	Limit states design
FIP	Federation Internationale de la précontrainte	LSP	Linear static procedure
FORM	First-order reliability method	LVDT	Linear Variable Differential Transformer
FOSM	First-order second-moment	LVL	Laminated veneer lumber
FPF	First-ply-failure	MAE	Mid-America Earthquake Center
FRC	Fiber-reinforced concrete	MCAA	Mason Contractors' Association of America
FRP	Fiber-reinforced polymer	MCE	Maximum considered earthquake
FVD	Fluid viscous damper	MDA	Market Development Association
GMAW	Gas metal arc welding	MDOF	Multi-degree-of-freedom
HAZ	Heat-affected zone	ME	Maximum earthquake
HDPE	High-density polyethylene	MIG	Metal arc inert gas welding
HOG	House over garage	MLIT	Ministry of Land, Infrastructure and Transport
HPC	High-performance concrete	MMI	Modified Mercalli Intensity
HPS	High-performance steel	MR	Magnetorheological
HSLA	High-strength low-alloy	MRF	Moment-resisting frame
HSS	Hollow structural section	MSE	Mechanically stabilized earth
HVAC	Heating, ventilating, and air conditioning	MSJC	Masonry Standards Joint Committee
IBC	International Building Code	MVFSM	Mean value first-order second-moment
ICBO	International Conference of Building Officials	NA	Nonlinear analysis
ICC	International Code Council	NAMC	North American Masonry Conference
IDA	Incremental dynamic analysis	NCMA	National Concrete Masonry Association
IDARC	Inelastic damage analysis of reinforced concrete structure	NDA	Nonlinear dynamic analysis
IDR	Interstory drift ratios		
IIW	International Institute of Welding		
ILSS	Interlamina shear strength		

NDE	Nondestructive evaluation	SBCC	Southern Building Code Congress
NDP	Non-linear dynamic procedure	SBCCI	Southern Building Code Congress International
NDS	National design specification	SCBF	Special concentrically braced frames
NEHRP	National Earthquake Hazard Reduction Program	SCC	Self-consolidation concrete
NESSUS	Numerical Evaluation of Stochastic Structures Under Stress	SCF	Stress concentration factor
NFPA	National Fire Prevention Association	SCL	Structural composite lumber
NLA	National Lime Association	SDAP	Seismic design and analysis procedure
NSM	Near-surface-mounted	SDC	Seismic design category
NSP	Non-linear static procedure	SDOF	Single degree-of-freedom
OCBF	Ordinary concentrically braced frames	SDR	Seismic design requirement
OMF	Ordinary moment frame	SE	Serviceable earthquake
OSB	Oriental strand board	SEAOC	Structural Engineers Association of California
PAAP	Practical advanced analysis program	SEAONC	Structural Engineers Association of Northern California
PBD	Performance-based design	SEE	Safety evaluation earthquake
PBSE	Performance-based seismic engineering	SFOBB	San Francisco-Oakland Bay Bridge
PCA	Portland Cement Association	SHRP	Strategic Highway Research Program
PCI	Prestressed Concrete Institute	SLS	Serviceable limit state
PD	Plastic design	SMAW	Shielded metal arc welding
PDF	Probability density function	SMF	Special moment frame
PE	Probability of exceedance	SOE	Support of excavation
PEER	Pacific Earthquake Engineering Research Center	SORM	Second-order reliability method
PEM	Pseudo-excitation method	SPDM	Structural Plastics Design Manual
PGA	Peak ground acceleration	SPL	Seismic performance level
PGD	Peak ground displacement	SRC	Steel and reinforced concrete
PGV	Peak ground velocity	SRF	Stiffness reduction factor
PI	Point of inflection	SRSS	Square-root-of-the-sum-of-the-squares
POF	Probability of failure	SSI	Soil-structure interaction
PPWS	Prefabricated parallel-wire strand	SSRC	Structured Stability Research Council
PROBAN	PROBability ANalysis	STMF	Special truss moment frame
PSV	Pseudospectral velocity	SUG	Seismic use group
PTI	Post-Tensioning Institute	TBM	Tunnel boring machine
PVC	Polyvinyl chloride	TCCMAR	Technical Coordinating Committee for Masonry Research
PWS	Parallel wire strand	TERECO	TEaching RELiability CONcepts
Q&T	Quenching and tempering	TIG	Tungsten arc inert gas welding
QST	Quenching and self-tempering process	TLD	Tuned liquid damper
RBS	Reduced beam section	TMCP	Thermal-mechanical controlled processing
RBSO	Reliability Based Structural Optimization	TMD	Tuned mass damper
RC	Reinforced concrete	TMS	The Masonry Society
RHS	Rectangular hollow section		
RMS	Root-mean-square		
SAW	Submerged arc welding		
SBC	Slotted bolted connection		
SBC	Standard Building Code		

TT	Through the thickness
UBC	Uniform Building Code
UDL	Uniformed distributed load
ULS	Ultimate limit state
URM	Unreinforced masonry
USDA	US Department of Agriculture
USGS	US Geological Survey
VE	Viscoelastic

VF	Viscous fluid
VRT	Variance reduction technique
WF	Wide flange
WRF	Wave reflection factor
WSMF	Welded special moment-frame
WUF-W	Welded-unreinforced flange, welded web
ZPA	Zero period acceleration

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I

Structural Analysis

1

Structural Fundamentals

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1.1 Stresses

1.1.1 Stress Components and Traction

Consider an infinitesimal parallelepiped element shown in Figure 1.1. The state of stress of this element is defined by nine stress components or tensors (σ_{11} , σ_{12} , σ_{13} , σ_{21} , σ_{22} , σ_{23} , σ_{31} , σ_{32} , and σ_{33}), of which six (σ_{11} , σ_{22} , σ_{33} , $\sigma_{12} = \sigma_{21}$, $\sigma_{23} = \sigma_{32}$, and $\sigma_{13} = \sigma_{31}$) are independent. The stress components that act normal to the planes of the parallelepiped (σ_{11} , σ_{22} , σ_{33}) are called normal stresses, and the stress components that act tangential to the planes of the parallelepiped ($\sigma_{12} = \sigma_{21}$, $\sigma_{23} = \sigma_{32}$, $\sigma_{13} = \sigma_{31}$) are called shear stresses. The first subscript of each stress component refers to the face on which the stress acts, and the second subscript refers to the direction in which the stress acts. Thus, σ_{ij} represents a stress acting on the i face in the j direction. A face is considered positive if a unit vector drawn *perpendicular* to the face directing outward from the inside of the element is pointing in the positive direction as defined

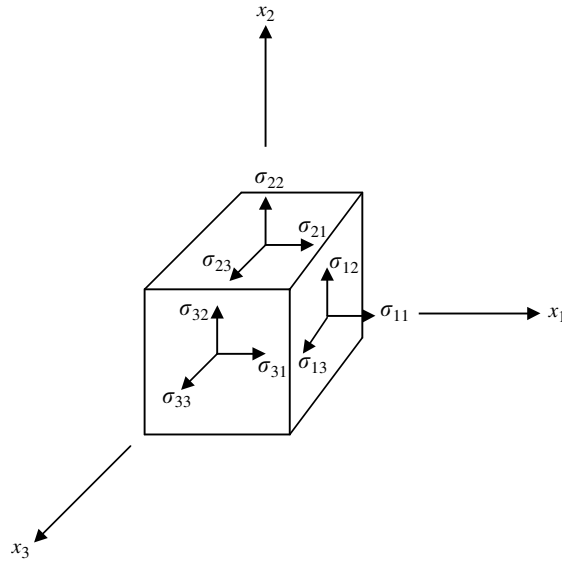


FIGURE 1.1 Stress components acting on the positive faces of a parallelepiped element.

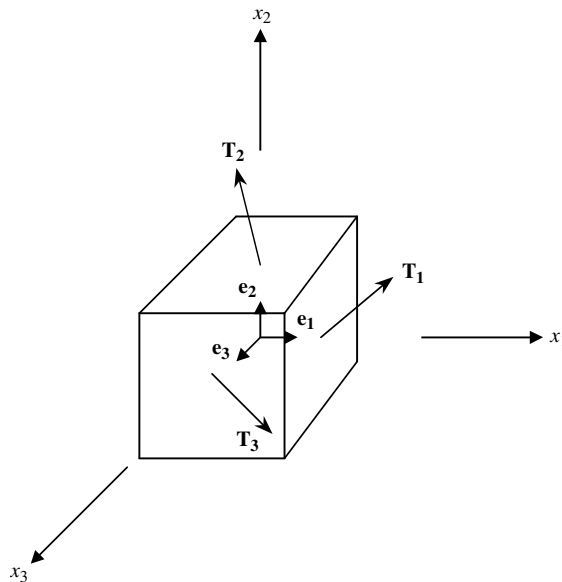


FIGURE 1.2 Tractions acting on the positive faces of a parallelepiped element.

by the Cartesian coordinate system (x_1, x_2, x_3) . A stress is considered positive if it acts on a positive face in the positive direction or if it acts on a negative face in the negative direction. It is considered negative if it acts on a positive face in the negative direction or if it acts on a negative face in the positive direction.

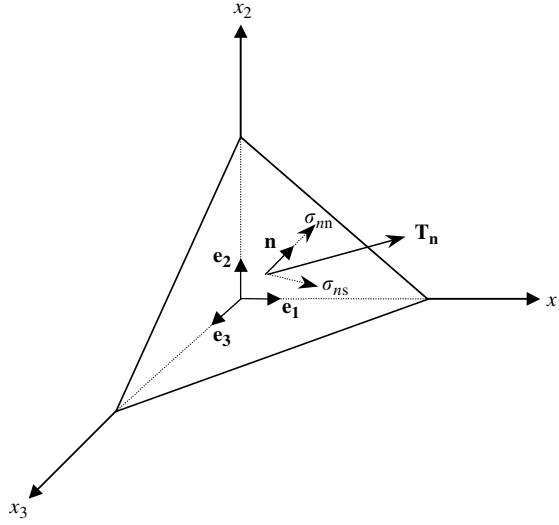


FIGURE 1.3 Traction and stresses acting on an arbitrary plane.

The vectorial sum of the three stress components acting on each face of the parallelepiped produces a traction \mathbf{T} . Thus, the tractions acting on the three positive faces of the element shown in Figure 1.2 are given by

$$\begin{aligned}\mathbf{T}_1 &= \sigma_{11}\mathbf{e}_1 + \sigma_{12}\mathbf{e}_2 + \sigma_{13}\mathbf{e}_3 \\ \mathbf{T}_2 &= \sigma_{21}\mathbf{e}_1 + \sigma_{22}\mathbf{e}_2 + \sigma_{23}\mathbf{e}_3 \\ \mathbf{T}_3 &= \sigma_{31}\mathbf{e}_1 + \sigma_{32}\mathbf{e}_2 + \sigma_{33}\mathbf{e}_3\end{aligned}\quad (1.1)$$

where \mathbf{e}_1 , \mathbf{e}_2 , and \mathbf{e}_3 are unit vectors corresponding to the x_1 , x_2 , and x_3 axes, respectively.

Equations 1.1 can be written in tensor or indicial notation as

$$\mathbf{T}_i = \sigma_{ij}\mathbf{e}_j \quad (1.2)$$

Note that both indices (i and j) range from 1 to 3. The dummy index (j in the above equation) denotes summation.

Using Cauchy's definition (Bathe 1982), traction is regarded as the intensity of a force resultant acting on an infinitesimal area. Mathematically, it is expressed as

$$\mathbf{T}_i = \frac{d\mathbf{F}_i}{dA_i} \quad (1.3)$$

1.1.2 Stress on an Arbitrary Surface

If the tractions acting on three orthogonal faces of a volume element are known, or calculated using Equations 1.1, the traction \mathbf{T}_n acting on any arbitrary surface as defined by a unit normal vector \mathbf{n} ($= n_1\mathbf{e}_1 + n_2\mathbf{e}_2 + n_3\mathbf{e}_3$) as shown in Figure 1.3 can be written as

$$\mathbf{T}_n = T_1\mathbf{e}_1 + T_2\mathbf{e}_2 + T_3\mathbf{e}_3 \quad (1.4)$$

where T_1 , T_2 , and T_3 are the components of \mathbf{T}_n acting in the 1, 2, and 3 directions, respectively, of the Cartesian coordinate system shown. They can be calculated using Cauchy's formulas:

$$\begin{aligned}T_1 &= \sigma_{11}n_1 + \sigma_{21}n_2 + \sigma_{31}n_3 \\ T_2 &= \sigma_{12}n_1 + \sigma_{22}n_2 + \sigma_{32}n_3 \\ T_3 &= \sigma_{13}n_1 + \sigma_{23}n_2 + \sigma_{33}n_3\end{aligned}\quad (1.5)$$

or using indicial notation:

$$T_i = \sigma_{ji} n_j \quad (1.6)$$

Once \mathbf{T}_n is known, the normal stress σ_{nn} and shear stress σ_{ns} acting on the arbitrary plane as defined by the unit vector \mathbf{n} can be calculated using the equations

$$\sigma_{nn} = \mathbf{T}_n \cdot \mathbf{n} = T_i n_i = T_1 n_1 + T_2 n_2 + T_3 n_3 \quad (1.7)$$

$$\sigma_{ns} = (T_i T_i - \sigma_{nn}^2)^{1/2} = (T_1^2 + T_2^2 + T_3^2 - \sigma_{nn}^2)^{1/2} \quad (1.8)$$

EXAMPLE 1.1

If the state of stress at a point in Cartesian coordinates is given by

$$\begin{bmatrix} 200 & -80 & 20 \\ -80 & 150 & 40 \\ 20 & 40 & -100 \end{bmatrix} \text{ MPa}$$

Determine:

1. The traction that acts on a plane with unit normal vector $\mathbf{n} = \frac{1}{2}\mathbf{e}_1 + \frac{1}{2}\mathbf{e}_2 + \frac{1}{\sqrt{2}}\mathbf{e}_3$
2. The normal stress and shear stress that act on this plane

Solution

1. The components of traction that act on the specified plane can be calculated using Equation 1.6:

$$\begin{aligned} T_1 &= \sigma_{11} n_1 + \sigma_{21} n_2 + \sigma_{31} n_3 \\ &= (200)\left(\frac{1}{2}\right) + (-80)\left(\frac{1}{2}\right) + (20)\left(\frac{1}{\sqrt{2}}\right) \\ &= 74.1 \text{ MPa} \\ T_2 &= \sigma_{12} n_1 + \sigma_{22} n_2 + \sigma_{32} n_3 \\ &= (-80)\left(\frac{1}{2}\right) + (150)\left(\frac{1}{2}\right) + (40)\left(\frac{1}{\sqrt{2}}\right) \\ &= 63.3 \text{ MPa} \\ T_3 &= \sigma_{13} n_1 + \sigma_{23} n_2 + \sigma_{33} n_3 \\ &= (20)\left(\frac{1}{2}\right) + (40)\left(\frac{1}{2}\right) + (-100)\left(\frac{1}{\sqrt{2}}\right) \\ &= -40.7 \text{ MPa} \end{aligned}$$

From Equation 1.4, the traction acting on the specified plane is

$$\mathbf{T}_n = 74.1\mathbf{e}_1 + 63.3\mathbf{e}_2 - 40.7\mathbf{e}_3$$

2. The normal and shear stresses acting on the plane can be calculated from Equations 1.7 and 1.8, respectively,

$$\begin{aligned} \sigma_{nn} &= T_1 n_1 + T_2 n_2 + T_3 n_3 \\ &= (74.1)\left(\frac{1}{2}\right) + (63.3)\left(\frac{1}{2}\right) + (-40.7)\left(\frac{1}{\sqrt{2}}\right) \\ &= 40 \text{ MPa} \\ \sigma_{ns} &= (T_1^2 + T_2^2 + T_3^2 - \sigma_{nn}^2)^{1/2} \\ &= \sqrt{(74.1)^2 + (63.3)^2 + (-40.7)^2 - (40)^2} \\ &= 97.7 \text{ MPa} \end{aligned}$$

1.1.3 Stress Transformation

If the state of stress acting on an infinitesimal volume element corresponding to a Cartesian coordinate system $(x_1 - x_2 - x_3)$ as shown in Figure 1.1 is known, the state of stress on the element with respect to another Cartesian coordinate system $(x'_1 - x'_2 - x'_3)$ can be calculated using the tensor equation

$$\sigma'_{ij} = l_{ik} l_{jl} \sigma_{kl} \quad (1.9)$$

where l is the direction cosine of two axes (one corresponding to the new and the other corresponding to the original). For instance,

$$l_{ik} = \cos(i', k), \quad l_{jl} = \cos(j', l) \quad (1.10)$$

represent the cosine of the angle formed by the new (i' or j') and the original (k or l) axes.

1.1.4 Principal Stresses and Principal Planes

Principal stresses are normal stresses that act on planes where the shear stresses are zero. Principal planes are planes on which principal stresses act. Principal stresses are calculated from the equation

$$\det \begin{vmatrix} \sigma_{11} - \sigma & \sigma_{12} & \sigma_{13} \\ \sigma_{12} & \sigma_{22} - \sigma & \sigma_{23} \\ \sigma_{13} & \sigma_{23} & \sigma_{33} - \sigma \end{vmatrix} = 0 \quad (1.11)$$

which, upon expansion, gives a cubic equation in σ :

$$\sigma^3 - I_1 \sigma^2 - I_2 \sigma - I_3 = 0 \quad (1.12)$$

where I_1 , I_2 , and I_3 are the first, second, and third stress invariants (their magnitudes remain unchanged regardless of the choice of the Cartesian coordinate axes) given by

$$\begin{aligned} I_1 &= \sigma_{11} + \sigma_{22} + \sigma_{33} \\ I_2 &= -\det \begin{vmatrix} \sigma_{11} & \sigma_{12} \\ \sigma_{12} & \sigma_{22} \end{vmatrix} - \det \begin{vmatrix} \sigma_{11} & \sigma_{13} \\ \sigma_{13} & \sigma_{33} \end{vmatrix} - \det \begin{vmatrix} \sigma_{22} & \sigma_{23} \\ \sigma_{23} & \sigma_{33} \end{vmatrix} \\ I_3 &= \det \begin{vmatrix} \sigma_{11} & \sigma_{12} & \sigma_{13} \\ \sigma_{12} & \sigma_{22} & \sigma_{23} \\ \sigma_{13} & \sigma_{23} & \sigma_{33} \end{vmatrix} \end{aligned} \quad (1.13)$$

The three roots of Equation 1.12, herein denoted as σ_{p1} , σ_{p2} , and σ_{p3} , are the principal stresses acting on the three orthogonal planes. The components of a unit vector that defines the principal plane (i.e., n_{1p_i} , n_{2p_i} , n_{3p_i}) corresponding to a specific principal stress σ_{p_i} (with $i = 1, 2, 3$) can be evaluated using any two of the following equations:

$$\begin{aligned} n_{1p_i}(\sigma_{11} - \sigma_{p_i}) + n_{2p_i}\sigma_{12} + n_{3p_i}\sigma_{13} &= 0 \\ n_{1p_i}\sigma_{12} + n_{2p_i}(\sigma_{22} - \sigma_{p_i}) + n_{3p_i}\sigma_{23} &= 0 \\ n_{1p_i}\sigma_{13} + n_{2p_i}\sigma_{23} + n_{3p_i}(\sigma_{33} - \sigma_{p_i}) &= 0 \end{aligned} \quad (1.14)$$

and

$$n_{1p_i}^2 + n_{2p_i}^2 + n_{3p_i}^2 = 1 \quad (1.15)$$

The unit vector calculated for each value of σ_{p_i} represents the direction of a *principal axis*. Thus, three principal axes that correspond to the three principal planes can be identified.

Note that the three stress invariants in Equations 1.13 can also be written in terms of the principal stresses:

$$\begin{aligned} I_1 &= \sigma_{p1} + \sigma_{p2} + \sigma_{p3} \\ I_2 &= -\sigma_{p1}\sigma_{p2} - \sigma_{p2}\sigma_{p3} - \sigma_{p1}\sigma_{p3} \\ I_3 &= \sigma_{p1}\sigma_{p2}\sigma_{p3} \end{aligned} \quad (1.16)$$

EXAMPLE 1.2

Suppose a plane stress condition exists, derive the equations for (1) stress transformation, (2) principal stresses, and (3) principal planes for this condition.

Solution

1. *Stress transformation.* With reference to Figure 1.4, a direct application of Equation 1.9, with the condition $\sigma_{33} = \sigma_{23} = \sigma_{13} = 0$ applying to a plane stress condition, gives the following stress transformation equations:

$$\begin{aligned} \sigma'_{11} &= \sigma_{11} \cos^2 \theta + \sigma_{22} \sin^2 \theta + \sigma_{12} \cos 2\theta \\ \sigma'_{22} &= \sigma_{11} \sin^2 \theta + \sigma_{22} \cos^2 \theta - \sigma_{12} \cos 2\theta \\ \sigma'_{12} &= \sigma_{11} \cos \theta \sin \theta + \sigma_{22} \sin \theta \cos \theta - \sigma_{12} \cos 2\theta \end{aligned}$$

Using the trigonometric identities

$$\cos(90^\circ - \theta) = \sin \theta, \quad \cos(90^\circ + \theta) = -\sin \theta,$$

$$\sin^2 \theta = \frac{1 - \cos 2\theta}{2}, \quad \cos^2 \theta = \frac{1 + \cos 2\theta}{2}, \quad \sin \theta \cos \theta = \frac{\sin 2\theta}{2}$$

the stress transformation equations can be expressed as

$$\begin{aligned} \sigma'_{11} &= \left(\frac{\sigma_{11} + \sigma_{22}}{2} \right) + \left(\frac{\sigma_{11} - \sigma_{22}}{2} \right) \cos 2\theta + \sigma_{12} \sin 2\theta \\ \sigma'_{22} &= \left(\frac{\sigma_{11} + \sigma_{22}}{2} \right) - \left(\frac{\sigma_{11} - \sigma_{22}}{2} \right) \cos 2\theta - \sigma_{12} \sin 2\theta \\ \sigma'_{12} &= -\left(\frac{\sigma_{11} - \sigma_{22}}{2} \right) \sin 2\theta + \sigma_{12} \cos 2\theta \end{aligned}$$

which are the familiar two-dimensional (2-D) stress transformation equations found in a number of introductory mechanics of materials books (see, e.g., Beer et al. 2001; Gere 2004).

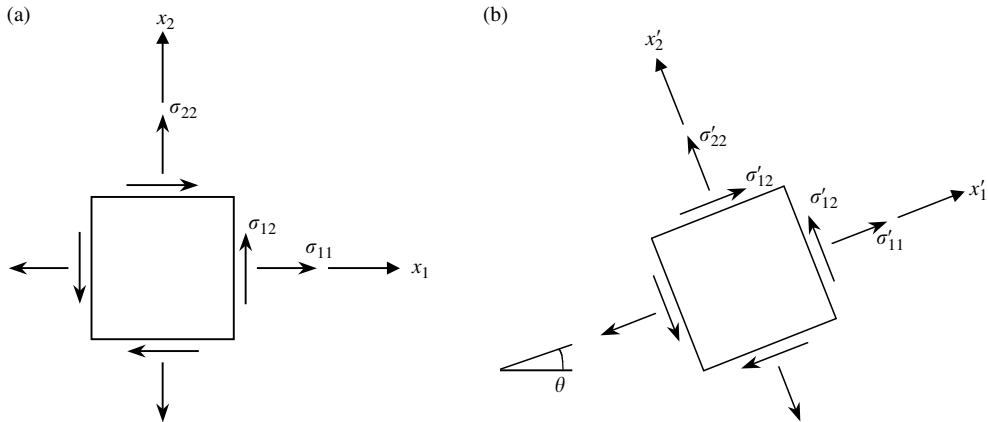


FIGURE 1.4 Two-dimensional (2-D) stress transformation: (a) original state of stress acting on a 2-D infinitesimal element and (b) transformed state of stress acting on a 2-D infinitesimal element.

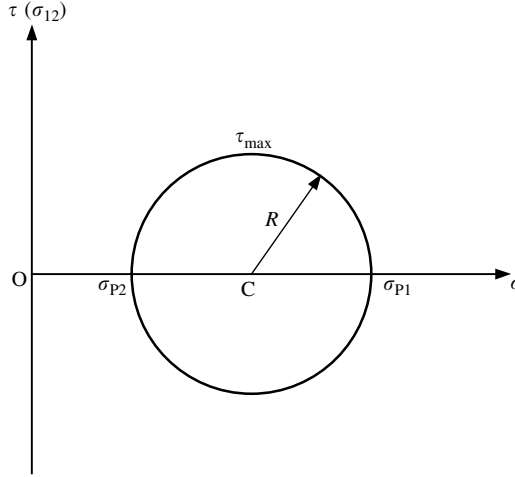


FIGURE 1.5 Mohr's circle.

2. *Principal stresses.* For plane stress condition, Equation 1.11 becomes

$$\det \begin{vmatrix} \sigma_{11} - \sigma & \sigma_{12} \\ \sigma_{12} & \sigma_{22} - \sigma \end{vmatrix}$$

which, upon expansion, gives the quadratic equation

$$\sigma^2 - (\sigma_{11} + \sigma_{22})\sigma + (\sigma_{11}\sigma_{22} - \sigma_{12}^2) = 0$$

The two solutions to the above equation give the two principal stresses as

$$\sigma_{P1} = \frac{\sigma_{11} + \sigma_{22}}{2} + \sqrt{\left(\frac{\sigma_{11} - \sigma_{22}}{2}\right)^2 + \sigma_{12}^2}$$

$$\sigma_{P2} = \frac{\sigma_{11} + \sigma_{22}}{2} - \sqrt{\left(\frac{\sigma_{11} - \sigma_{22}}{2}\right)^2 + \sigma_{12}^2}$$

Note that these stresses represent the rightmost and leftmost points on a *Mohr circle* (Beer et al. 2001), shown in Figure 1.5, with $OC = (\sigma_{11} + \sigma_{22})/2$ and $R = \sqrt{[(\sigma_{11} - \sigma_{22})/2]^2 + \sigma_{12}^2}$. (Although not asked for in this example, it can readily be seen that the maximum shear stress is the uppermost point on the Mohr circle given by $\tau_{\max} = (\sigma_{12})_{\max} = R = \sqrt{[(\sigma_{11} - \sigma_{22})/2]^2 + \sigma_{12}^2}$.)

3. *Principal planes.* Substituting the equation for σ_{P1} into

$$n_{1P1}(\sigma_{11} - \sigma) + n_{2P1}\sigma_{12} = 0$$

and recognizing that

$$n_{1P1}^2 + n_{2P1}^2 = 1$$

it can be shown that the principal plane on which σ_{P1} acts forms an angle $\theta_{P1} = \tan^{-1}(n_{2P1}/n_{1P1})$ with the x_1 (or x) axis and is given by

$$\theta_{P1} = \frac{1}{2} \tan^{-1} \left[\frac{\sigma_{12}}{(\sigma_{11} - \sigma_{22})/2} \right]$$

Following the same procedure for σ_{p2} or, more conveniently, by realizing that the two principal planes are orthogonal to each other, we have

$$\theta_{p2} = \theta_{p1} + \frac{\pi}{2}$$

(Note that the planes on which the maximum shear stress acts make an angle of $\pm 45^\circ$ with the principal planes, that is, $\theta_{s1} = \theta_{p1} - (\pi/4)$, $\theta_{s2} = \theta_{p2} - (\pi/4) = \theta_{p1} + (\pi/4)$.)

1.1.5 Octahedral, Mean, and Deviatoric Stresses

Octahedral normal and shear stresses are stresses that act on planes with direction indices satisfying the condition $n_1^2 = n_2^2 = n_3^2 = \frac{1}{3}$ with respect to the three *principal axes* of an infinitesimal volume element. Since there are eight such planes, which together form an octahedron, the stresses acting on these planes are referred to as octahedral stresses. The equations for the octahedral normal and shear stresses are given by

$$\begin{aligned}\sigma_{\text{oct}} &= \frac{1}{3} I_1 \\ \tau_{\text{oct}} &= \frac{1}{3} \sqrt{2I_1^2 + 6I_2}\end{aligned}\quad (1.17)$$

where I_1 and I_2 are the first and second stress invariants defined in Equations 1.13 or in Equations 1.16. Octahedral stresses are used to define certain failure criteria (e.g., von Mises) for ductile materials.

Mean stress is obtained as the arithmetic average of three normal stresses (or the three principal stresses):

$$\sigma_m = \frac{1}{3}(\sigma_{11} + \sigma_{22} + \sigma_{33}) = \frac{1}{3}(\sigma_{p1} + \sigma_{p2} + \sigma_{p3}) = \frac{1}{3}I_1 \quad (1.18)$$

Deviatoric stress is defined by the stress tensor

$$\begin{bmatrix} \frac{2\sigma_{11} - \sigma_{22} - \sigma_{33}}{3} & \sigma_{12} & \sigma_{13} \\ \sigma_{12} & \frac{2\sigma_{22} - \sigma_{11} - \sigma_{33}}{3} & \sigma_{23} \\ \sigma_{13} & \sigma_{23} & \frac{2\sigma_{33} - \sigma_{11} - \sigma_{22}}{3} \end{bmatrix} \quad (1.19)$$

The deviatoric stress tensor represents a state of *pure shear*. It is obtained by subtracting the mean stress from the three normal stresses (σ_{11} , σ_{22} , and σ_{33}) in a stress tensor. It is important from the viewpoint of inelastic analysis because experiments have shown that inelastic behavior of most ductile materials is independent of the mean normal stress, but is related primarily to the deviatoric stress.

If the indicial notation s_{ij} is used to represent the nine deviatoric stress components given in Equation 1.19, the maximum deviatoric stress acting on each of the three orthogonal planes (which are the same as the principal planes) can be computed from the cubic equation

$$s^3 - J_1 s^2 - J_2 s - J_3 = 0 \quad (1.20)$$

where J_1 , J_2 , and J_3 are the first, second, and third deviatoric stress invariants given by

$$\begin{aligned}J_1 &= s_{ii} = s_{11} + s_{22} + s_{33} = 0 \\ J_2 &= \frac{1}{2}s_{ij}s_{ji} = \frac{1}{2}(s_{11}^2 + s_{22}^2 + s_{33}^2 + 2s_{12}^2 + 2s_{23}^2 + 2s_{13}^2) \\ J_3 &= \frac{1}{3}s_{ij}s_{jk}s_{ki} = \det \begin{vmatrix} s_{11} & s_{12} & s_{13} \\ s_{12} & s_{22} & s_{23} \\ s_{13} & s_{23} & s_{33} \end{vmatrix}\end{aligned}\quad (1.21)$$

Alternatively, if the principal stresses are known, the three maximum deviatoric stresses can be calculated using the equations

$$\begin{aligned}s_{p1} &= \frac{(\sigma_{p1} - \sigma_{p3}) + (\sigma_{p1} - \sigma_{p2})}{3} \\s_{p2} &= \frac{(\sigma_{p2} - \sigma_{p3}) + (\sigma_{p2} - \sigma_{p1})}{3} \\s_{p3} &= \frac{(\sigma_{p3} - \sigma_{p1}) + (\sigma_{p3} - \sigma_{p2})}{3}\end{aligned}\quad (1.22)$$

Note that J_1 , J_2 , and J_3 can also be expressed in terms of I_1 , I_2 , and I_3 , or the three maximum deviatoric stresses, as follows:

$$\begin{aligned}J_1 &= s_{p1} + s_{p2} + s_{p3} = 0 \\J_2 &= I_2 + \frac{1}{3}I_1^2 = \frac{1}{2}(s_{p1}^2 + s_{p2}^2 + s_{p3}^2) \\J_3 &= I_3 + \frac{1}{3}I_1 I_2 + \frac{2}{27}I_1^3 = \frac{1}{3}(s_{p1}^2 + s_{p2}^2 + s_{p3}^2) = s_{p1}s_{p2}s_{p3}\end{aligned}\quad (1.23)$$

1.1.6 Maximum Shear Stresses

If the principal stresses are known, the maximum shear stresses that act on each of the three orthogonal planes, which bisect the angle between the principal planes with direction indices ($n_1 = \pm 1/\sqrt{2}$, $n_2 = \pm 1/\sqrt{2}$, $n_3 = 0$), ($n_1 = 0$, $n_2 = \pm 1/\sqrt{2}$, $n_3 = \pm 1/\sqrt{2}$), ($n_1 = \pm 1/\sqrt{2}$, $n_2 = 0$, $n_3 = \pm 1/\sqrt{2}$) with respect to the principal axes, are given by

$$\begin{aligned}\tau_{\max 1} &= \frac{1}{2}|\sigma_{p1} - \sigma_{p2}| \\ \tau_{\max 2} &= \frac{1}{2}|\sigma_{p2} - \sigma_{p3}| \\ \tau_{\max 3} &= \frac{1}{2}|\sigma_{p1} - \sigma_{p3}|\end{aligned}\quad (1.24)$$

Note that the planes (called principal shear planes) on which these stresses act are *not* pure shear planes. The corresponding normal stresses that act on these principal shear planes are $(\sigma_{p1} + \sigma_{p2})/2$, $(\sigma_{p2} + \sigma_{p3})/2$, and $(\sigma_{p1} + \sigma_{p3})/2$, respectively.

1.2 Strains

1.2.1 Strain Components

Corresponding to the six stress components described in the preceding section are six strain components. With reference to a Cartesian coordinate system with axes labeled 1, 2, and 3 as in Figure 1.1, these strains are denoted as ε_{11} , ε_{22} , ε_{33} , $\gamma_{12} = 2\varepsilon_{12}$, $\gamma_{23} = 2\varepsilon_{23}$, and $\gamma_{31} = 2\varepsilon_{31}$. ε_{11} , ε_{22} , and ε_{33} are called normal strains and γ_{12} , γ_{23} , and γ_{31} are called shear strains. Using the definitions for *engineering* strains (Bathe 1982), normal strain is defined as the ratio of the change in length to the original length of a straight line element, and shear strain is defined as the change in angle (when the element is in a strained state) from an originally right angle (when the element is in an unstrained state).

1.2.2 Strain–Displacement Relationships

If we denote u , v , and w as the translational displacements in the 1, 2, and 3 (or x , y , and z) directions, respectively, then according to the small-displacement theory the six engineering strain components can be written in terms of these displacements as

$$\begin{aligned}\varepsilon_{11} &= \frac{\partial u}{\partial x}, \quad \varepsilon_{22} = \frac{\partial v}{\partial y}, \quad \varepsilon_{33} = \frac{\partial w}{\partial z}, \\ \gamma_{12} = 2\varepsilon_{12} &= \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}, \quad \gamma_{23} = 2\varepsilon_{23} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y}, \quad \gamma_{31} = 2\varepsilon_{31} = \frac{\partial w}{\partial x} + \frac{\partial u}{\partial z}\end{aligned}\quad (1.25)$$

1.2.3 Strain Analysis

Like stresses, strains can be transformed from one Cartesian coordinate system to another. This can be done by replacing σ by ε in Equation 1.9. In addition, one can calculate the three principal strains (ε_{p1} , ε_{p2} , and ε_{p3}) and the maximum shear strains ($\varepsilon_{\max 1}$, $\varepsilon_{\max 2}$, and $\varepsilon_{\max 3}$) acting on the three orthogonal planes by following the same procedure outlined in the preceding section for stresses simply by replacing all occurrences of σ by ε in Equations 1.12 and 1.24. However, it should be noted that except for isotropic elastic materials, principal planes for stresses and principal planes for strains do not necessarily coincide, nor do the planes of maximum shear stresses and maximum shear strains.

1.3 Equilibrium and Compatibility

By using an infinitesimal parallelepiped element subject to a system of positive three-dimensional stresses, equilibrium of the element requires that the following three equations relating the stresses be satisfied (Wang 1953; Timoshenko and Goodier 1970):

$$\begin{aligned}\frac{\partial \sigma_{11}}{\partial x} + \frac{\partial \sigma_{12}}{\partial y} + \frac{\partial \sigma_{13}}{\partial z} + B_x &= 0 \\ \frac{\partial \sigma_{12}}{\partial x} + \frac{\partial \sigma_{22}}{\partial y} + \frac{\partial \sigma_{23}}{\partial z} + B_y &= 0 \\ \frac{\partial \sigma_{13}}{\partial x} + \frac{\partial \sigma_{23}}{\partial y} + \frac{\partial \sigma_{33}}{\partial z} + B_z &= 0\end{aligned}\tag{1.26}$$

where B_x , B_y , and B_z are the body forces per unit volume acting in the 1, 2, and 3 (or x , y , and z) directions, respectively.

According to Equations 1.25, the six strain components can be expressed in terms of just three displacement variables (u , v , and w). To obtain a unique solution for the displacements for a given loading condition, these strains must be related. By manipulating Equations 1.25, it can be shown (Wang 1953; Timoshenko and Goodier 1970) that the strains are related by the following compatibility equations:

$$\begin{aligned}\frac{\partial^2 \varepsilon_{11}}{\partial y^2} + \frac{\partial^2 \varepsilon_{22}}{\partial x^2} &= 2 \frac{\partial^2 \varepsilon_{12}}{\partial x \partial y} \\ \frac{\partial^2 \varepsilon_{22}}{\partial z^2} + \frac{\partial^2 \varepsilon_{33}}{\partial y^2} &= 2 \frac{\partial^2 \varepsilon_{23}}{\partial y \partial z} \\ \frac{\partial^2 \varepsilon_{11}}{\partial z^2} + \frac{\partial^2 \varepsilon_{33}}{\partial x^2} &= 2 \frac{\partial^2 \varepsilon_{13}}{\partial x \partial z} \\ \frac{\partial^2 \varepsilon_{11}}{\partial y \partial z} &= -\frac{\partial^2 \varepsilon_{23}}{\partial x^2} + \frac{\partial^2 \varepsilon_{13}}{\partial x \partial y} + \frac{\partial^2 \varepsilon_{12}}{\partial x \partial z} \\ \frac{\partial^2 \varepsilon_{22}}{\partial x \partial z} &= \frac{\partial^2 \varepsilon_{23}}{\partial x \partial y} - \frac{\partial^2 \varepsilon_{13}}{\partial y^2} + \frac{\partial^2 \varepsilon_{12}}{\partial y \partial z} \\ \frac{\partial^2 \varepsilon_{33}}{\partial x \partial y} &= \frac{\partial^2 \varepsilon_{23}}{\partial x \partial z} + \frac{\partial^2 \varepsilon_{13}}{\partial y \partial z} - \frac{\partial^2 \varepsilon_{12}}{\partial z^2}\end{aligned}\tag{1.27}$$

Since Equations 1.27 were derived from Equations 1.25, they should not be regarded as an independent set of equations. The six stress components (σ_{11} , σ_{22} , σ_{33} , σ_{12} , σ_{23} , and σ_{13}), the six strain components (ε_{11} , ε_{22} , ε_{33} , ε_{12} , ε_{23} , and ε_{13}), and the three displacement components (u , v , and w) constitute a total of 15 unknowns, which cannot be solved using the three equilibrium equations (Equations 1.26) and the six compatibility equations (Equations 1.27). To do so, six additional equations are needed. These equations, which relate stresses with strains, are described in the next section.

1.4 Stress–Strain Relationship

Stress–strain (or constitutive) relationship defines how a material behaves when subjected to applied loads. Depending on the type of material and the magnitude of the applied loads, a material may behave elastically or inelastically. A material is said to behave elastically when loading and unloading follow the same path and no permanent deformation occurs upon full unloading (see Figure 1.6, Paths 1 and 2). A material is said to behave inelastically when loading and unloading do not follow the same path and permanent deformation results upon full unloading (see Figure 1.6, Paths 1 and 3). A material that behaves elastically may be further classified as linear or nonlinear, depending on whether Paths 1 and 2 in Figure 1.6 are linear or nonlinear. If the properties of a material are independent of location in the material, the material is said to be homogenous. Moreover, depending on the directional effect of the mechanical properties exhibited by a material, terms such as isotropic, orthotropic, monoclinic, or anisotropic can also be used to describe a material.

1.4.1 Linear Elastic Behavior

If the material is anisotropic (i.e., no plane of symmetry exists for the material properties), the six stress components are related to the six strain components by 21 independent material constants (D_{ij} in the following matrix equation):

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{12} \\ \sigma_{23} \\ \sigma_{13} \end{Bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & D_{14} & D_{15} & D_{16} \\ & D_{22} & D_{23} & D_{24} & D_{25} & D_{26} \\ & & D_{33} & D_{34} & D_{35} & D_{36} \\ & & & D_{44} & D_{45} & D_{46} \\ & \text{sym.} & & & D_{55} & D_{56} \\ & & & & & D_{66} \end{bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{23} \\ \varepsilon_{13} \end{Bmatrix} \quad (1.28)$$

If the material is monoclinic (i.e., material properties are symmetric about one plane), the number of independent material constants reduces to 13. For instance, if the plane defined by the x_2 – x_3 (or y – z)

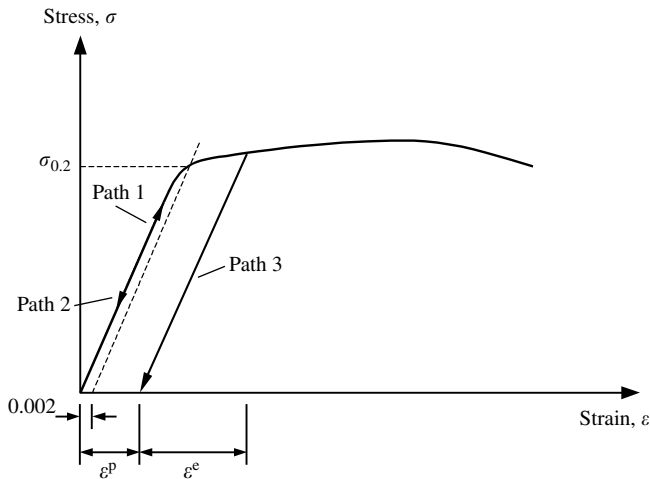


FIGURE 1.6 Uniaxial stress–strain curve.

axes is the plane of symmetry, the stress–strain relationship takes the form

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{12} \\ \sigma_{23} \\ \sigma_{13} \end{Bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & D_{15} & 0 \\ & D_{22} & D_{23} & 0 & D_{25} & 0 \\ & & D_{33} & 0 & D_{35} & 0 \\ & & & D_{44} & 0 & D_{46} \\ & \text{sym.} & & & D_{55} & 0 \\ & & & & & D_{66} \end{bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{23} \\ \varepsilon_{13} \end{Bmatrix} \quad (1.29)$$

If the material is orthotropic (i.e., material properties are symmetric about two planes), the number of independent material constants further reduces to 9, and the stress–strain relationship takes the form

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{12} \\ \sigma_{23} \\ \sigma_{13} \end{Bmatrix} = \begin{bmatrix} D_{11} & D_{12} & D_{13} & 0 & 0 & 0 \\ & D_{22} & D_{23} & 0 & 0 & 0 \\ & & D_{33} & 0 & 0 & 0 \\ & & & D_{44} & 0 & 0 \\ & \text{sym.} & & & D_{55} & 0 \\ & & & & & D_{66} \end{bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{23} \\ \varepsilon_{13} \end{Bmatrix} \quad (1.30)$$

If the material is isotropic (i.e., material properties are independent of the direction), the number of independent material constants becomes 2:

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{12} \\ \sigma_{23} \\ \sigma_{13} \end{Bmatrix} = \begin{bmatrix} 2\mu + \lambda & \lambda & \lambda & 0 & 0 & 0 \\ & 2\mu + \lambda & \lambda & 0 & 0 & 0 \\ & & 2\mu + \lambda & 0 & 0 & 0 \\ & & & 2\mu & 0 & 0 \\ & \text{sym.} & & & 2\mu & 0 \\ & & & & & 2\mu \end{bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{23} \\ \varepsilon_{13} \end{Bmatrix} \quad (1.31)$$

where μ and λ are called Lamé constants. They are related to the elastic modulus E and Poisson's ratio ν of the material by the following equations:

$$\mu = \frac{E}{2(1 + \nu)} \quad (1.32)$$

$$\lambda = \frac{\nu E}{(1 + \nu)(1 - 2\nu)} \quad (1.33)$$

Note that $\mu = G$, the shear modulus of the material.

Regardless of the material type, experimental means are often needed to determine the material constants that relate the stresses and strains in Equations 1.28 to 1.31. Because of the difficulty in determining a large number of constants, analyses are often performed by assuming the material is either isotropic or orthotropic.

If we denote any of the above equations relating stresses and strains symbolically as

$$\boldsymbol{\sigma} = \mathbf{D}\boldsymbol{\varepsilon} \quad (1.34)$$

where $\boldsymbol{\sigma}$ is the 6×1 vector of stresses, $\boldsymbol{\varepsilon}$ is the 6×1 vector of strains, and \mathbf{D} is the 6×6 material stiffness matrix, it can be shown that

$$\boldsymbol{\varepsilon} = \mathbf{D}^{-1}\boldsymbol{\sigma} = \mathbf{C}\boldsymbol{\sigma} \quad (1.35)$$

where \mathbf{C} is the material compliance matrix. For an orthotropic material, the expanded form of Equation 1.35 is

$$\begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{23} \\ \varepsilon_{13} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_{11}} & \frac{-\nu_{21}}{E_{22}} & \frac{-\nu_{31}}{E_{33}} & 0 & 0 & 0 \\ & \frac{1}{E_{22}} & \frac{-\nu_{32}}{E_{33}} & 0 & 0 & 0 \\ & & \frac{1}{E_{33}} & 0 & 0 & 0 \\ & & & \frac{1}{2G_{12}} & 0 & 0 \\ & \text{sym.} & & & \frac{1}{2G_{23}} & 0 \\ & & & & & \frac{1}{2G_{13}} \end{bmatrix} \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{12} \\ \sigma_{23} \\ \sigma_{13} \end{Bmatrix} \quad (1.36)$$

where E_{11} , E_{22} , and E_{33} denote the orthotropic moduli of elasticity measured in three orthogonal directions, G_{12} , G_{23} , and G_{13} denote the orthotropic shear moduli, and ν_{ij} denotes the Poisson's ratio obtained by dividing the negative value of the strain induced in the j direction by the strain produced in the i direction by a stress applied in the i direction.

For an isotropic material, the expanded form of Equation 1.35 is

$$\begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{23} \\ \varepsilon_{13} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E} & \frac{-\nu}{E} & \frac{-\nu}{E} & 0 & 0 & 0 \\ & \frac{1}{E} & \frac{-\nu}{E} & 0 & 0 & 0 \\ & & \frac{1}{E} & 0 & 0 & 0 \\ & & & \frac{(1+\nu)}{E} & 0 & 0 \\ & \text{sym.} & & & \frac{(1+\nu)}{E} & 0 \\ & & & & & \frac{(1+\nu)}{E} \end{bmatrix} \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{12} \\ \sigma_{23} \\ \sigma_{13} \end{Bmatrix} \quad (1.37)$$

where the first three of the above matrix equation are often referred to as generalized Hooke's Law for linear elastic, homogeneous, and isotropic materials, respectively.

EXAMPLE 1.3

Determine the stress-strain relationship for a homogeneous isotropic material assuming (1) plane stress condition and (2) plane strain condition.

Solution

1. *Plane stress condition.* If the stresses are acting on the x_1 - x_2 (or x - y) plane, plane stress condition implies that $\sigma_{33} = \sigma_{23} = \sigma_{13} = 0$. Substituting this condition into Equation 1.37, we have

$$\begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{12} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E} & \frac{-\nu}{E} & 0 \\ & \frac{1}{E} & 0 \\ \text{sym.} & & \frac{1+\nu}{E} \end{bmatrix} \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{Bmatrix} \quad \text{and} \quad \varepsilon_{33} = -\frac{\nu}{E}(\sigma_{11} + \sigma_{22})$$

or

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{Bmatrix} = \begin{bmatrix} \frac{E}{1-\nu^2} & \frac{\nu E}{1-\nu^2} & 0 \\ & \frac{E}{1-\nu^2} & 0 \\ \text{sym.} & & \frac{E}{1+\nu} \end{bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{12} \end{Bmatrix} \quad \text{and} \quad \varepsilon_{33} = \frac{-\nu}{1-\nu} (\varepsilon_{11} + \varepsilon_{22})$$

Note that $\varepsilon_{33} \neq 0$ even though $\sigma_{33} = 0$ (i.e., a biaxial state of stress gives rise to a triaxial state of strain) because of the *Poisson's effect*.

2. *Plane strain condition.* If the strain is negligible in the x_3 (or z) direction, plane strain condition implies that $\varepsilon_{33} = \varepsilon_{23} = \varepsilon_{13} = 0$. Substituting this condition into Equation 1.31, we have

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{Bmatrix} = \begin{bmatrix} \frac{(1-\nu)E}{1-\nu-2\nu^2} & \frac{\nu E}{1-\nu-2\nu^2} & 0 \\ & \frac{(1-\nu)E}{1-\nu-2\nu^2} & 0 \\ \text{sym.} & & \frac{E}{1+\nu} \end{bmatrix} \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{12} \end{Bmatrix} \quad \text{and} \quad \sigma_{33} = \frac{\nu E}{1-\nu-2\nu^2} (\varepsilon_{11} + \varepsilon_{22})$$

or

$$\begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{12} \end{Bmatrix} = \begin{bmatrix} \frac{1-\nu^2}{E} & \frac{-\nu(1+\nu)}{E} & 0 \\ & \frac{1-\nu^2}{E} & 0 \\ \text{sym.} & & \frac{1+\nu}{E} \end{bmatrix} \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{Bmatrix} \quad \text{and} \quad \sigma_{33} = \nu(\sigma_{11} + \sigma_{22})$$

Note that $\sigma_{33} \neq 0$ even though $\varepsilon_{33} = 0$.

1.4.2 Nonlinear Elastic Behavior

If an elastic material exhibits nonlinear behavior, the stress-strain relationship is often cast in *incremental form* relating some increments of strains to stress, or vice versa

$$d\sigma = D_I d\varepsilon \quad (1.38)$$

or

$$d\varepsilon = C_I d\sigma \quad (1.39)$$

where $d\sigma$ is the incremental vector of stresses, $d\varepsilon$ is the incremental vector of strains, D_I is the incremental material stiffness matrix, and C_I is the incremental material compliance matrix. If the experimental stress-strain curves of a material are known, the terms in these matrices can be taken as the values of the tangential or secant slopes of these curves. The analysis of structures made of materials that exhibit nonlinear elastic behavior has to be performed numerically in incremental steps as well.

Alternatively, if the nonlinear relationship between any given components of stress (or strain) can be expressed as a mathematical function of strains (or stresses) and material constants k_1, k_2, k_3 , etc., as follows:

$$\sigma_{ij} = f_{ij}(\varepsilon_{11}, \varepsilon_{22}, \varepsilon_{33}, \varepsilon_{12}, \varepsilon_{23}, \varepsilon_{13}, k_1, k_2, k_3, \dots) \quad (1.40)$$

$$\varepsilon_{ij} = g_{ij}(\sigma_{11}, \sigma_{22}, \sigma_{33}, \sigma_{12}, \sigma_{23}, \sigma_{13}, k_1, k_2, k_3, \dots) \quad (1.41)$$

such relationships can be incorporated directly into the analysis to obtain closed-form solutions. However, this type of analysis can be performed only if both the structure and the loading conditions are very simple.

EXAMPLE 1.4

Derive the load–deflection equation for the axially loaded member shown in Figure 1.7. The member is made from a material with a uniaxial stress–strain relationship described by the equation $\varepsilon = B(\sigma/BnE_0)^n$, where B and n are material constants and E_0 is the initial slope of the stress–strain curve (i.e., the slope at $\sigma = 0$).

The deflection (which for this problem is equal to the elongation) of the axially loaded member can be obtained by integrating the strain over the length of the member; that is,

$$\begin{aligned}\delta &= \int_0^L \varepsilon \, dx = \int_0^L B \left(\frac{\sigma}{BnE_0} \right)^n \, dx = \int_0^L B \left[\frac{P}{BnE_0A_0 \left(1 - \frac{x}{2L} \right)} \right]^n \, dx \\ &= \left(\frac{P}{nE_0A_0} \right)^n \left(\frac{2^n - 2}{n - 1} \right) B^{1-n} L\end{aligned}$$

1.4.3 Inelastic Behavior

For structures subject to uniaxial loading, inelastic behavior occurs once the stress in the structure exceeds the yield stress, σ_y , of the material. The yield stress is defined as the stress beyond which inelastic or permanent strain is induced, as shown in Figure 1.6. While some materials (e.g., structural steel) exhibit a definitive yield point on the uniaxial stress–strain curve, others do not. For such cases, the yield stress is often determined graphically using the 0.2% offset method. In this method, a line parallel to the initial slope of the uniaxial stress–strain curve is drawn from the 0.2% strain point. The 0.2% yield stress is obtained as the stress at which this line intersects the stress–strain curve.

For structures subject to biaxial or triaxial loading, inelastic behavior is assumed to occur when some combined stress state reaches a yield envelope (for a 2-D problem) or a yield surface (for a 3-D problem). Mathematically, the yield condition can be expressed as

$$f(\sigma_{ij}, k_1, k_2, k_3, \dots) = 0 \quad (1.42)$$

where k_1, k_2, k_3, \dots are (experimentally determined) material constants.

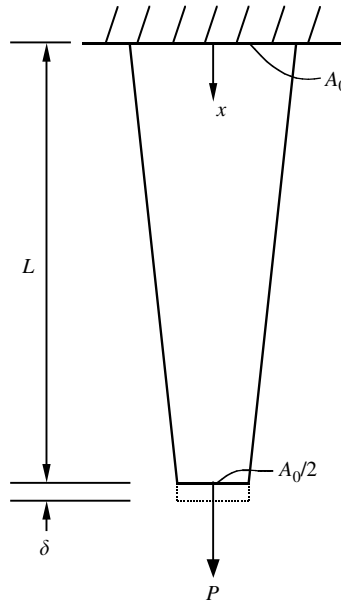


FIGURE 1.7 Tapered axially loaded member.

Over the years, various yield functions f have been proposed to describe the yield condition of a variety of materials (see, e.g., Chen 1982; Chen and Baladi 1985; Chakrabarty 1987; Chen and Han 1988). For ductile materials (e.g., most metals), the Tresca and von Mises yield criteria are often used. A brief discussion of these two criteria is given below:

1. *Tresca criterion.* According to the Tresca yield criterion, yielding occurs when the maximum shear stress at a point calculated using Equations 1.24 reaches a critical value equal to $\sigma_y/2$, where σ_y is the yield stress of the material obtained from a simple tension test. Mathematically, the Tresca yield criterion is expressed as

$$\max \left\{ \begin{array}{l} \frac{1}{2} |\sigma_{P1} - \sigma_{P2}| \\ \frac{1}{2} |\sigma_{P2} - \sigma_{P3}| \\ \frac{1}{2} |\sigma_{P1} - \sigma_{P3}| \end{array} \right\} = \frac{\sigma_y}{2} \quad (1.43)$$

2. *von Mises criterion.* Despite its simplicity, one drawback of the Tresca yield criterion is that it does not take into consideration the effect of the intermediate principal stress. One method to include the effect of this principal stress in the yield function is to use the octahedral shearing stress (or the strain energy of distortion) as the key parameter to describe yielding in the materials. The von Mises yield criterion is one example. The von Mises yield criterion has the form

$$\left[\frac{(\sigma_{P1} - \sigma_{P2})^2 + (\sigma_{P2} - \sigma_{P3})^2 + (\sigma_{P1} - \sigma_{P3})^2}{6} \right]^{1/2} = \frac{\sigma_y}{\sqrt{3}} \quad (1.44)$$

where σ_y is the yield stress obtained from a simple tension test.

It should be noted that both the Tresca and the von Mises yield criteria are independent of hydrostatic pressure effect. As a result, they should be used only for materials that are pressure insensitive. For pressure dependent materials (e.g., soils), other yield (or failure) criteria should be used. A few of these criteria are given below:

1. *Rankine criterion.* This criterion is often used to describe the tensile (fracture) failure of a brittle material. It has the form

$$\sigma_{P1} = \sigma_u, \quad \sigma_{P2} = \sigma_u, \quad \sigma_{P3} = \sigma_u \quad (1.45)$$

where σ_u is the ultimate (or tensile) strength of the material. For materials that exhibit brittle behavior in tension, but ductile behavior in confined compression (e.g., concrete, rocks, and soils), the Rankine criterion is sometimes combined with the Tresca or von Mises criterion to describe the failure behavior of the materials. If used in this context, the criterion is referred to as the Tresca or von Mises criterion with a tension cut-off.

2. *Mohr–Coulomb criterion.* This criterion is often used to describe the shear failure of soil. Failure is said to occur when a limiting shear stress reaches a value as defined by an envelope, which is expressed as a function of normal stress, soil cohesion, and friction angle. If the principal stresses are such that $\sigma_{P1} > \sigma_{P2} > \sigma_{P3}$, the Mohr–Coulomb criterion can be written as

$$\frac{1}{2}(\sigma_{P1} - \sigma_{P3}) \cos \phi = c - \left[\frac{1}{2}(\sigma_{P1} + \sigma_{P3}) + \frac{\sigma_{P1} - \sigma_{P3}}{2} \sin \phi \right] \tan \phi \quad (1.46)$$

where c is the cohesion and ϕ is the angle of internal friction.

3. *Drucker–Prager criterion.* This criterion is an extension of the von Mises criterion, where the influence of hydrostatic stress on failure is incorporated by the addition of the term αI_1 , where

I_1 is the first stress invariant as defined in Equations 1.13 (note that $\sigma_{11} + \sigma_{22} + \sigma_{33} = \sigma_{P1} + \sigma_{P2} + \sigma_{P3}$)

$$\alpha(\sigma_{P1} + \sigma_{P2} + \sigma_{P3}) + \left[\frac{(\sigma_{P1} - \sigma_{P2})^2 + (\sigma_{P2} - \sigma_{P3})^2 + (\sigma_{P1} - \sigma_{P3})^2}{6} \right]^{1/2} = k \quad (1.47)$$

where α and k are material constants to be determined by curve-fitting of the above equation to experimental data.

If yielding does not signify failure of a material (which is often the case for ductile materials), the postyield behavior of the material is described by the use of a flow rule. A flow rule establishes the relative magnitudes of the components of plastic strain increment $d\epsilon_{ij}^p$ and the direction of the plastic strain increment in the strain space. It is written as

$$d\epsilon_{ij}^p = d\lambda \frac{\partial g}{\partial \sigma_{ij}} \quad (1.48)$$

where $d\lambda$ is a positive scalar factor of proportionality, g is a plastic potential in stress space, and $\partial g / \partial \sigma_{ij}$ is the gradient, which represents the direction of a normal vector to the surface defined by the plastic potential at point σ_{ij} . Equation 1.48 implies that $d\epsilon_{ij}^p$ is directed along the normal to the surface of the plastic potential. If the plastic potential g is equal to the yield function f , Equation 1.48 is called the associated flow rule. Otherwise, it is called the nonassociated flow rule.

Using the elastic stress–strain relationship expressed in Equation 1.39, the flow rule expressed in Equation 1.48 with $g=f$ (i.e., associated flow rule), the *consistency condition* for an elastic–perfectly plastic material given by

$$df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} = 0 \quad (1.49)$$

and the following relationship among total, elastic, and inelastic (plastic) strains,

$$d\epsilon_{ij} = d\epsilon_{ij}^e + d\epsilon_{ij}^p \quad (1.50)$$

it has been shown (Chen and Han 1988) that an incremental stress–strain relationship for an elastic–perfectly plastic material that follows the associated flow rule can be written as

$$d\sigma_{ij} = D_{ijkl}^{ep} d\epsilon_{kl} \quad (1.51)$$

where D_{ijkl}^{ep} is the incremental elastic–perfectly plastic material stiffness matrix given by

$$D_{ijkl}^{ep} = D_{ijkl} - \frac{D_{ijmn}(\partial f / \partial \sigma_{mn})(\partial f / \partial \sigma_{pq})D_{pqkl}}{(\partial f / \partial \sigma_{rs})D_{rstu}(\partial f / \partial \sigma_{tu})} \quad (1.52)$$

where D_{ijkl} (or D_{ijmm} , D_{pqkb} , etc.) is the indicial form of \mathbf{D}_I given in Equation 1.38.

1.4.4 Hardening Rules

If a material exhibits *work-hardening* behavior in which a state of stress beyond yield can exist, then in addition to the initial yield surface f a new yield surface, called subsequent yield or loading surface F , needs to be defined. Like the initial yield surface, the loading surface demarcates elastic behavior from inelastic behavior. If the stress point moves on or within the loading surface, no additional plastic strain will be induced. If the stress point is on the loading surface and the loading condition is such that it pushes the stress point out of the loading surface, additional plastic deformations will occur. When this happens, the configuration of the loading surface will change. The condition of loading and unloading for a multiaxial stress state is mathematically defined as follows.

If the stress point is on the loading surface (i.e., if $F=0$), loading occurs if

$$n_{ij}^F d\sigma_{ij} > 0 \quad (1.53)$$

and unloading occurs if

$$n_{ij}^F d\sigma_{ij} < 0 \quad (1.54)$$

where n_{ij}^F represents a component of a unit vector that is normal to the loading surface F , that is,

$$n_{ij}^F = \frac{\partial F / \partial \sigma_{ij}}{\sqrt{(\partial F / \partial \sigma_{kl})(\partial F / \partial \sigma_{kl})}} \quad (1.55)$$

For the special case when $n_{ij}^F d\sigma_{ij} = 0$, that is, the loading vector $d\sigma_{ij}$ is perpendicular to the corresponding component of the unit normal vector n_{ij}^F , a state of neutral loading is said to have occurred. Note that additional plastic strain is induced only during loading, but not during neutral loading or unloading.

According to the incremental or flow theory of plasticity, the configuration of the loading surface when loading occurs can be described by the use of a hardening rule. A hardening rule establishes a relationship between the subsequent yield stress of a material and the inelastic deformation accumulated during prior excursion into the inelastic regime. A number of hardening rules have been proposed over the years. They can often be classified into or associated with one of the following:

1. *Isotropic hardening.* This hardening rule assumes that during plastic deformations, the loading surface is merely an expansion, without distortion, of the initial yield surface. Mathematically, this surface is represented by the equation

$$F(\sigma_{ij}) = k^2(\epsilon_p) \quad (1.56)$$

where k is a constant, which is a function of the total (i.e., cumulated) plastic strain ϵ_p . Although this is one of the simplest hardening rules, it has a serious drawback in that it cannot be used to account for the *Bauschinger effect*, which states that the occurrence of an initial plastic deformation in one direction (e.g., in tension) will cause a reduction in material resistance to a subsequent plastic deformation in the opposite direction (e.g., in compression). Since the Bauschinger effect is present in most structural materials, the use of isotropic hardening should be limited to problems that involve only *monotonic loading* in which no stress reversals will occur.

2. *Kinematic hardening.* This hardening rule (Prager 1955, 1956) assumes that during plastic deformation, the loading surface is formed by a simple rigid body translation (with no change in size, shape, and orientation) of the initial yield surface in stress space. Thus, the equation of the loading surface takes the form

$$F(\sigma_{ij} - \eta_{ij}) = k^2 \quad (1.57)$$

where k is a constant to be determined experimentally and η_{ij} are the coordinates of the centroid of the loading surface, which changes continuously throughout plastic deformation. It should be noted that contrary to isotropic hardening, kinematic hardening takes full account of the Bauschinger effect, so much so that the amount of “loss” of material resistance in one direction during subsequent plastic deformation is exactly equal to the amount of initial plastic deformation the material experiences in the opposite direction, which may or may not be truly reflective of real material behavior.

3. *Mixed hardening.* As the name implies, this hardening rule (Hodge 1957) contains features of both the isotropic and the kinematic hardening rules described above. It has the form

$$F(\sigma_{ij} - \eta_{ij}) = k^2(\epsilon_p) \quad (1.58)$$

where η_{ij} and k are as defined in Equations 1.56 and 1.57. In mixed hardening, the loading surface is defined by a translation (as described by the term η_{ij}) and expansion (as measured by the term $k(\epsilon_p)$), but no change in shape, of the initial yield surface. The advantage of using the mixed hardening rule is that one can conveniently simulate different degrees of the Bauschinger effect by adjusting the two hardening parameters (η_{ij} and k) of the model.

1.4.5 Effective Stress and Effective Plastic Strain

Effective stress and effective plastic strain are variables that allow the hardening parameters contained in the above hardening models to be correlated with an experimentally obtained uniaxial stress–strain curve of the material. The effective stress has unit of stress, and it should reduce to the stress σ_{11} in a uniaxial stress condition. Table 1.1 summarizes the equations for the effective stress and hardening parameter for two materials modeled using the isotropic hardening rule. The equations shown in Table 1.1 can also be used for materials modeled using the kinematic or mixed hardening rule provided that the effective stress σ_e is replaced by a reduced effective stress σ_e^r , computed using a reduced stress tensor given by

$$\sigma_{ij}^r = \sigma_{ij} - \eta_{ij} \quad (1.59)$$

Effective plastic strain increment $d\epsilon_e^p$ can be defined in the context of plastic work per unit volume in the form

$$dW_p = \sigma_e d\epsilon_e^p \quad (1.60)$$

By using Equation 1.48 in conjunction with a material model, it can be shown (Chen and Han 1988) that for a von Mises material

$$d\epsilon_e^p = \sqrt{\frac{2}{3}} d\epsilon_{ij}^p d\epsilon_{ij}^p \quad (1.61)$$

and for a Drucker–Prager material

$$d\epsilon_e^p = \frac{\alpha + (1/\sqrt{3})}{\sqrt{3\alpha^2 + (1/2)}} \sqrt{d\epsilon_{ij}^p d\epsilon_{ij}^p} \quad (1.62)$$

The effective stress and effective plastic strain are related by the incremental stress–strain equation

$$d\sigma_e = H_p d\epsilon_e^p \quad (1.63)$$

where H_p is the plastic modulus, which is obtained as the slope of the uniaxial stress–plastic strain curve at the current value of σ_e .

Using the concept of effective plastic strain, flow rule, consistency condition, relationship between total, elastic, and plastic strains, elastic stress–strain relationship, and a hardening rule, it can be shown (Chen and Han 1988) that an incremental stress–strain relationship for an elastic–work-hardening material can be written in the form of Equation 1.51 with

$$D_{ijkl}^{ep} = D_{ijkl} - \frac{D_{ijmn}(\partial g / \partial \sigma_{mn})(\partial F / \partial \sigma_{pq})D_{pqkl}}{\kappa + (\partial F / \partial \sigma_{rs})D_{rstu}(\partial g / \partial \sigma_{tu})} \quad (1.64)$$

TABLE 1.1 Effective Stress

Material model	Effective stress, σ_e	Hardening parameter, k
von Mises	$\sqrt{3J_2}$	$\sigma_e/\sqrt{3}$
Drucker–Prager	$(\sqrt{3}\alpha I_1 + \sqrt{3J_2})/(1 + \sqrt{3}\alpha)$	$(\alpha + (1/\sqrt{3}))\sigma_e$

Note: J_2 is the second deviatoric stress invariant defined in Equations 1.21, I_1 is the first stress invariant defined in Equations 1.13, and α is a material constant defined in Equation 1.47.

where

$$\kappa = -\frac{\partial F}{\partial \varepsilon_{ij}^p} \frac{\partial g}{\partial \sigma_{ij}} - \frac{\partial F}{\partial k} \frac{dk}{d\varepsilon_e^p} C \sqrt{\frac{\partial g}{\partial \sigma_{ij}} \frac{\partial g}{\partial \sigma_{ij}}} \quad (1.65)$$

where C is a material constant, which is equal to

$$\sqrt{\frac{2}{3}} \quad (1.66a)$$

for a von Mises material and

$$\frac{\alpha + (1/\sqrt{3})}{\sqrt{3\alpha^2 + (1/2)}} \quad (1.66b)$$

for a Drucker–Prager material. From Equation 1.64 it can be seen that D_{ijkl}^{ep} is not necessarily symmetric unless the associated flow rule (i.e., $g = F$) is used in the formulation.

1.5 Stress Resultants

Structural analysis can be performed and results represented in terms of stresses and strains, or forces and displacements. For skeletal structures (i.e., structures that are made up of line elements such as trusses, beams, frames, arches, grillages, etc.), the internal forces and moments, or stress resultants, acting on a given cross-section as shown in Figure 1.8 are related to the stresses acting over the cross-section by the following equations:

$$\begin{aligned} F_x &= \int_A \sigma_{11} dA, & F_y &= \int_A \sigma_{12} dA, & F_z &= \int_A \sigma_{13} dA \\ M_x &= \int_A (-\sigma_{12}z + \sigma_{13}y) dA, & M_y &= \int_A \sigma_{11}z dA, & M_z &= -\int_A \sigma_{11}y dA \end{aligned} \quad (1.67)$$

where F_x is the axial force, F_y and F_z are the shear forces, M_x is the torque, and M_y and M_z are the bending moments about the y (or x_2) and z (or x_3) axes, respectively. Note that the value of some of these terms

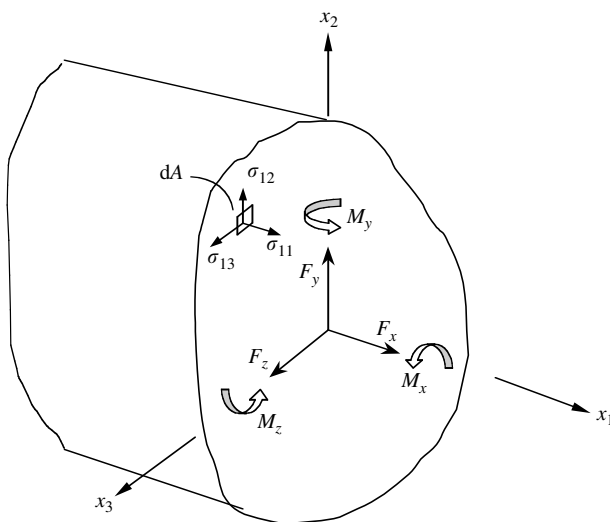


FIGURE 1.8 Stress resultants.

is 0 depending on the structure and the assumptions used in the modeling. For instance, in a truss analysis, it is often assumed that $F_y = F_z = M_x = M_y = M_z = 0$. In a 2-D beam–frame analysis in which the structure is modeled on the x – y (or x_1 – x_2) plane, it is often assumed that $F_z = M_x = M_y = 0$. In a 2-D grillage analysis in which the structure is modeled on the x – z (or x_1 – x_3) plane, it is often assumed that $F_x = F_z = M_y = 0$.

1.6 Types of Analyses

Depending on the magnitude of the applied loads, the type of structure under consideration, the purpose of performing the analysis, and the degree of accuracy desired, different types of analyses can be performed to determine the force–displacement or stress–strain response of a structural system. Given below is a succinct discussion of some salient features associated with several types of analyses that one can perform depending on the objectives of the analysis and the expectations of the analyst. A more detailed discussion of some of these methods of analysis can be found in later chapters of this handbook.

1.6.1 First-Order versus Second-Order Analysis

A first-order analysis is one in which all equilibrium and kinematic equations are written with respect to the initial or undeformed configuration of the structure. A second-order analysis is one in which equilibrium and kinematic equations are written with respect to the current or deformed geometry of the structure. Because all structures deform under loads, a method of analysis that takes into consideration structural deformation in its formulation will provide a more realistic representation of the structure. However, because of its simplicity, a first-order analysis is often performed in lieu of a second-order analysis. Although the results obtained lack the precision of a second-order analysis, they are sufficiently accurate for design purpose if deflections or deformations of the structure are small.

1.6.2 Elastic versus Inelastic Analysis

An elastic analysis is one in which the effect of yielding is ignored in the analysis. Thus, the stress–strain relationships discussed in Section 1.4.1 (for linear elastic material behavior) or Section 1.4.2 (for nonlinear elastic material behavior) will be used in the analysis. Because all strains (and deformations) are recoverable in an elastic analysis, no consideration is given to the loading history or loading path dependent effect (which is very important in an inelastic analysis) during the analysis. Elastic analysis is therefore much easier to perform than inelastic analysis. However, if yielding does occur, a behavioral model that is capable of capturing the inelastic response of the structure should be used.

1.6.3 Plastic Hinge versus Plastic Zone Analysis

For framed structures, if the applied loads are proportional and monotonic, the loading history effect is inconsequential, and a plastic hinge (also called concentrated plasticity) or plastic zone (also called distributed plasticity) analysis can be performed to capture the inelastic behavior of the system. In the plastic hinge method (ASCE-WRC 1971) of analysis, inelasticity is assumed to concentrate in regions of plastic hinges. A plastic hinge is a zero-length element where the moment is equal to the cross-section plastic moment capacity M_p . If the effects of shear and axial force are ignored, M_p is given by

$$M_p = Z\sigma_y \quad (1.68)$$

where Z is the plastic section modulus (AISC 2001) and σ_y is the material yield stress.

In a simple plastic hinge analysis, once the moment in a cross-section reaches M_p , a hinge is inserted at that location and no additional moment is assumed to be carried by that cross-section. Cross-sections that have moments below M_p are assumed to behave elastically. Because the formation

of a plastic hinge is a gradual process in which yielding spreads slowly from the neutral axis toward the extreme fiber of the cross-section (i.e., cross-section plastification effect) as well as along the length of the member (i.e., member plastification effect) as the applied load increases, more realistic models such as the modified plastic hinge approach (White and Chen 1993) and the plastic zone approach (Vogel 1984, 1985; Lui and Zhang 1990; Clarke et al. 1992) have been proposed to capture this spread of plasticity effect. While the modified plastic hinge approach only accounts for cross-section plastification, the plastic zone approach accounts for both cross-section and member plastification as well as for the effect of residual stresses, and is therefore considered the most accurate method of frame analysis. Unfortunately, to achieve this high degree of accuracy, very careful and detailed modeling is required. For practical reasons, plastic zone analysis is rarely performed on a routine basis. It is mostly used as a research tool to calibrate or verify the accuracy of advanced in-house structural analysis programs.

1.6.4 Stability Analysis

Stability analysis is a special type of second-order analysis in which the system under consideration is subjected to compressive force or stress (Allen and Bulson 1980; Chen and Lui 1987, 1991; Bazant and Cedolin 1991). If the force or stress is high enough, a phenomenon known as instability or buckling may occur. At the buckling or critical load, the structural system loses its stiffness, changes its deformation pattern, and loses its ability to carry the applied loads. The mathematics used for the computation of this critical load is called an eigenvalue problem. The system critical load and buckled mode shape are obtained as the lowest eigenvalue and the corresponding eigenvector of the equation

$$\mathbf{K}\mathbf{U} = \lambda\mathbf{K}_G\mathbf{U} \quad (1.69)$$

where \mathbf{K} is the first-order system stiffness matrix, \mathbf{K}_G is the system geometrical stiffness matrix, λ is the eigenvalue of the system, and \mathbf{U} is the system displacement vector.

Stability analysis can be elastic or inelastic, depending on whether the stiffness matrices in Equation 1.69 are formulated assuming elastic or inelastic material behavior (McGuire et al. 2000). In addition, it should be noted that not all systems experience instability in the form of sudden buckling. Structural systems that are geometrically imperfect (which is often the case for real structures) undergo deformations that may resemble the buckled mode shapes at the outset of loading. The critical load for these geometrically imperfect systems is called the limit load. It is obtained as the peak point of the load–deflection curve generated using a second-order analysis.

1.6.5 Static versus Dynamics Analysis

A static analysis is one in which the effects of damping and inertia are not important and are therefore ignored. It is used when the loads acting on the structure are stationary or applied very slowly over time. A dynamic analysis is performed if the applied loads are time dependent, or if the effects of damping and

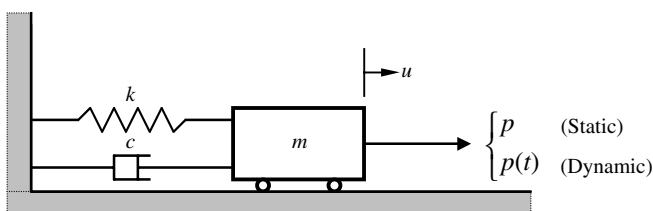


FIGURE 1.9 A simple spring–mass–damper system.

inertia are important. As shown in Figure 1.9, if a static analysis is used, the equilibrium equation has the form

$$ku = p \quad (1.70)$$

where k is the spring stiffness, u is the horizontal displacement of the mass, and p is the applied force. However, if a dynamic analysis is used, the equilibrium equation has the form

$$m\ddot{u} + c\dot{u} + ku = p(t) \quad (1.71)$$

where m is the mass, c is the damping coefficient, k is the spring stiffness, u is the displacement, \dot{u} is the velocity, \ddot{u} is the acceleration of the mass, and $p(t)$ is the time-varying applied load. $m\ddot{u}$ is called the inertia force, $c\dot{u}$ is the damping force from a viscous damper, and ku is the spring force. Note that inertia force and damping force are not present in the static equation, but they are present in the dynamic equation. It is also noteworthy to observe that the static equation is an algebraic equation, but the dynamic equation is a differential equation. A dynamic analysis is therefore more difficult and time consuming to perform than a static analysis, and depending on the form and complexity of the excitation function $p(t)$, recourse to numerical methods is often needed (Cheng 2001; Chopra 2001).

The system shown in Figure 1.9 is referred to as a single degree-of-freedom (dof) system because one displacement variable u is all that is needed to define the displaced configuration of the system. For a multiple dof system, Equations 1.70 and 1.71 need to be written in matrix form as

$$\mathbf{K}\mathbf{U} = \mathbf{P} \quad (1.72)$$

$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{C}\dot{\mathbf{U}} + \mathbf{K}\mathbf{U} = \mathbf{P}(\mathbf{t}) \quad (1.73)$$

where \mathbf{K} , \mathbf{M} , and \mathbf{C} are the system stiffness, system mass, and system damping matrices, respectively, \mathbf{U} , $\dot{\mathbf{U}}$, and $\ddot{\mathbf{U}}$ are the system displacement, velocity, and acceleration vectors, respectively, and $\mathbf{P}(\mathbf{t})$ is the time dependent system excitation force vector.

1.7 Structural Analysis and Design

Structural analysis refers to the computation of internal forces, displacements, stresses, and strains of a structure with known geometry, arrangement of components as well as component and material properties under a set of applied loads. Structural design refers to the determination of the proper material, geometry, arrangement of components and component properties to carry a predefined set of applied loads. In general, analysis and design are intertwined, and have to be performed iteratively in sequence. Using a preliminary set of structural and component geometry determined based on experience or the use of simplified behavioral models, an analysis is performed from which internal forces, displacements, stresses, and strains are calculated. These computed quantities are then used (often in conjunction with a design specification) to modify the preliminary design. The basic condition to satisfy in a strength based design is that

$$\text{capacity} \geq \text{demand} \quad (1.74)$$

Another analysis (called reanalysis) is then performed to obtain a more refined set of design quantities. The process is repeated until Equation 1.74 is satisfied in every part of the structure. Very often, different load combinations and different patterns of load applications have to be investigated to identify the worst possible scenario for design. As a result, the use of computers becomes indispensable for the design of complex structures.

Glossary

Mohr's circle — When plotted in a Cartesian coordinate system with the normal stress σ as the abscissa and the shear stress τ as the ordinate, a Mohr circle is a graphical representation of the state of

stress at a point. Each pair of coordinates on a Mohr circle represents the magnitude of a pair of normal and shear stresses that exist on a plane with a certain orientation.

Monotonic loading — A loading that does not change direction during the course of the load history.

Poisson's effect — An effect in which an increase (or decrease) of strain in one direction causes a decrease (or increase) of strains in other directions. It is quantified by what is referred to as Poisson's ratio ν , which is defined as the ratio of the minus value of the lateral strain to the longitudinal strain. Most materials have Poisson's ratios that fall in the range $0 < \nu \leq 0.5$.

Principal axes — The three orthogonal axes that are collinear with the unit vectors used to define the three principal planes of a parallelepiped volume element. Principal axes can also be defined as axes about which the product of inertia I_{ij} (when $i \neq j$) vanishes.

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Thin-Walled Structures, edited by J. Rhodes, J. Loughlan, and K.P. Chong, is an international journal that publishes papers on theory, experiment, design, etc. related to cold-formed steel sections, plate and shell structures, and others. It is published by Elsevier Applied Science. A special issue of the journal on cold-formed steel structures was edited by J. Rhodes and W.W. Yu, guest editors, and published in 1993.

Proceedings of the International Specialty Conference on Cold-Formed Steel Structures, edited by W.W. Yu, J.H. Senne, and R.A. LaBoube, have been published by the University of Missouri-Rolla since 1971. These publications contain technical papers presented at the International Specialty Conferences on Cold-Formed Steel Structures.

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FIGURE 18.91 Examples of beams in moment-resisting frames
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For further study of continuum analogy method of space frames, *Analysis and Design of Space Frames*

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The quarterly journal International Journal of Space Structures reports advances in the theory and

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b Tb required nodal torsional brace stiffness including web distortion

D translational displacement

D o column initial out-of-straightness

D os column out-of-straightness due to shortening

D sh shortening or compression element D T total column sway deflection f resistance factor γ complementary angle between diagonal brace and axial member or twist of member cross-section t inelastic stiffness reduction factor of

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a % system redundancy measure; reliability index
deterioration rate without maintenance

a c % condition index deterioration rate

b % reliability index

F % the distribution function of the standard normal variate

g % redundancy range; immediate improvement in reliability index after application of preventive maintenance $Z \geq 0.5$ % amount of damage when there is a 50% probability of detection m % mean value n % corrosion rate y % angle; reliability index deterioration rate during preventive maintenance effect Y % random variable r % correlation coefficient s % standard deviation of

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Appendix

WWW Sites

Nowadays the internet can provide a rich source of information on everything. There are many web

sites that give information on fire related topics. The following web sites are well-known organizations

that have a interest in the topic of this chapter, structural fire engineering. They are divided into three

groups: government and research organizations whose main functions are legislation, research, and

dissemination; academic institutions whose main functions are research and education; and industrial

companies that are involved in research and development to some extent, but whose main interest is

in the application of fire engineering in practical projects. It is inevitable that this list is biased toward

U.K. organizations.

A: Government and Research Organizations

www.bre.co.uk: Building Research Establishment (BRE), U.K.

www.safety.odpm.gov.uk/fire: Office of the Deputy Prime Minister, U.K.

www.nrc.ca: National Research Council of Canada (NRCC), Canada

www.bfnl.nist.org: Building and Fire Research Lab, National Institution of Science and Technology

(NIST), U.S.

www.nfpa.org: National Fire Protection Association (NFPA), U.S.

www.sfpe.org: Society of Fire Protection Engineers (SFPE), U.S.

www.iafss.org: International Association for Fire Safety Science (IAFSS)

www.vtt.fi: VTT Building Technology, Finland

www.sp.se: Swedish National Testing and Research Institute (SP), Sweden

www.sintef.no: Norwegian Fire Research Laboratory (SINTEF), Norway

www.factorymutual.com: Factory Mutual, U.S.

www.cticm.fr: Centre Technique Industriel de la Construction Mé'tallique (CTICM), France

www.tno.bouw.nl: TNO Building and Construction Research, The Netherlands

B: Academic institutions

www.structuralfiresafety.com: One-stop-shop for structural fire safety, set up by Manchester Centre for

Civil and Construction Engineering, University of Manchester Institute of Science and Technology

(UMIST), U.K.

www.steelinfire.org.uk: Steel In Fire Forum (STIFF), a network coordinated by the University of

Sheffield, U.K.

www.ed.ac.uk: University of Edinburgh, U.K.

www.ulst.ac.uk: University of Ulster, U.K.

www.brand.lth.se: Lund University, Sweden

www.wpi.edu: Worcester Polytechnic Institute, U.S.

www.enfp.umd.edu: University of Maryland, U.S.

www.civil.canterbury.ac.nz: University of Canterbury, New Zealand

www.vut.edu.au: Victoria University of Technology, Australia

C: Industrial Companies

www.corusgroup.com: U.K. steel manufacturer

www.arup.com: Ove Arup & Partners, Engineering consultancy, U.K.

www.burohappold.com: Buro Happold Consulting Engineers, Engineering Consultancy, U.K.

www.steel-sci.org: The Steel Construction Institute, U.K. of